The Investigation and Design of the Ord River Dam

BY K. C. WEBSTER, M.I.E.AUST. and W. J. WILKIN, B.C.E., M.I.E.AUST.*

Summary.—The Ord River Dam, which is a 320-ft-high rockfill dam with thin sloping clay core, was constructed between 1969 and 1971. The construction of a dam across the Ord River, which has a probable maximum flood of 2 500 000 cusecs, posed some unusual design problems.

The paper describes the investigation and design of the dam, the techniques adopted to allow flood waters to pass over the partially completed embankment and the development of an economic spillway arrangement.

INTRODUCTION

The Ord River Dam is the major engineering feature in the development of the Ord Irrigation Project. The dam forms a storage reservoir of 4 600 000 acre-ft capacity to spillway crest level. Water released from this storage will flow 30 miles down the Ord River to the Diversion Dam where it will be diverted into earth channels to supply some 180 000 acres of irrigable black-soil plains.

In 1941, Mr R. J. Dumas, Director of Works, and Mr F. Foreman, the Government Geologist, made the first engineering reconnaissance survey for a major dam on the Ord River. Six sites were identified in gorges where the Ord River cuts through the Carr Boyd Ranges. The site first favoured was surveyed and some preliminary drilling was carried out between 1942 and 1947. However, as the full magnitude of wet season discharges of the Ord River became apparent, the provision of adequate spillway capacity became a major problem and this, coupled with the depth to foundation rock at the first site, led to investigations being commenced in 1960 at a second site some three-quarters of a mile further downstream. Investigation and design proceeded at this site as resources permitted and, in November 1967, the Commonwealth made a grant of 20.9 million dollars to the State of Western Australia for the construction of the Ord River Dam. Construction of the dam commenced in 1969 and was substantially completed by December 1971.

DESCRIPTION OF WORKS

The works, shown in Fig. 2, comprise:

(*a*) A rockfill dam, some 1100 ft long at crest level and 320 ft high above the lowest foundation.

(*b*) An unlined spillway cut through a granite saddle approximately five miles from the dam embankment—the maximum depth of cut is 90 ft, the bottom width 40 ft and the length 7000 ft.

(*c*) Three auxiliary spillways through saddles in the north-east rim of the reservoir.

(*d*) An intake structure with two fixed-wheel gates, stop logs, trash racks and access bridge.

(*e*) Two conduits, mainly in tunnel, in the right abutment, each with an effective diameter of 14.5 ft and an approximate length of 1200 ft. The downstream ends of the tunnels are steel-lined and the irrigation outlet branch terminates in three steel pipes.

(*f*) Three irrigation outlets, each with a 78″-dia outlet valve and a 96″-dia guard valve to control the flow of irrigation water.

Provision is made for connection of the tunnels to a hydro-electric station to be constructed later.

The reservoir contains 4 200 000 acre-ft of active storage below full supply level and 27 700 000 acre-ft of flood storage. At full supply level the reservoir area is 280 square miles and at design flood level, 850 square miles.

The storage capacity was determined by the most economic combination of spillway and embankment and will produce a yield of 1 500 000 acre-ft per annum, well in excess of the irrigation demand.

GEOLOGY

The geological setting is a sequence of Upper Proterozoic sedimentary rocks showing only minor metamorphic effects. Rock types are principally quartzites, sandstones, siltstones and mudstones. Within the river bed there are extensive recent deposits of silts, sands and gravels, which are subject to periodic removal by high river flows. Auger drilling and jetting showed that these alluvial deposits were up to 100 ft deep.

Faulting in the area is common and intense local deformation has been caused by the major faults, of which there are three within the vicinity of the dam site (Fig. 3).

Jointing systems are well-developed and the rocks vary from thin-bedded and closely-jointed to massive. Soil covers parts of the site but intermittent scree and talus slopes are common. Generally, surface weathering is shallow except in fault zones and in the stronger joint planes.

The foundations of the dam are on both quartzite and siltstone. The left abutment is a quartzite ridge on the limits of an anticlinal fold. Erosion of this anticline has exposed siltstone on the downstream side of the ridge but a massive, extensively-jointed quartzite rampart remains as a capping on the upstream side. The rock of the right abutment rises steeply from the river to form a cliff of massive quartzites and sandstones. Above the cliff is a series of terraces, scree slopes and rocky outcrops.

SITE INVESTIGATION

Site investigation commenced in 1960 and continued intermittently until 1968. Subsurface exploration consisted of diamond drilling, water jetting and probing, trenching and the driving of two adits. A total of 52 diamond drill holes was put down, ranging in length from 13 to 301 ft, with a total length of approximately 7040 ft. Most of the early holes were cored at AXT or AX, whereas the later holes were generally NMLC or BMLC size.

Due to the hardness and the abrasiveness of the quartzite rock, the bit life in 1960-61 was 5.3 ft per bit. This was improved to 7.0 ft per bit in 1964 by the use of step bits and drilling oil. Core recoveries averaged 97% for holes drilled mainly in the massive quartzite. Poorer recoveries were experienced in the thinly-bedded silty quartzite, the figure being 84%. The two adits were driven in 1961 into the left abutment to test the thickness of the quartzite capping over the siltstone and to investigate the nature of the contact zone.

During the early years of the investigation, the climatic conditions, remoteness and ruggedness of the area posed considerable difficulties of access, communication and the provision of supplies.

HYDROLOGIC CONDITIONS

Catchment and Climate:

The Ord River Dam catchment is within the 18″ to 25″ rainfall belt and has an annual evaporation of 102″. It extends over an area of 17 800 square miles of mostly rough, hilly country with thin soils, rocky outcrops and with sparse cover of spinifex and small trees. There are two distinct seasons, a wet season and a dry season. The wet season extends from about December to April and the rains of this season occur somewhat irregularly from monsoons and the occasional tropical cyclone.

Rainfall and Streamflow:

Gauging of the river commenced in 1945 and has enabled estimation of runoff to be made from rainfall records back to the 1904-5 season. Flood peaks of around one million cusecs have occurred three times in the past 15 years. During such floods the river transports large volumes of sediment. Base flows are very small in relation to these flood flows and become insignificant or cease altogether for several months during the dry season.

There is wide variation in flows from year to year, as shown by the following figures which are based on synthesised flows over a 62-year period:

Lower decile of annual flows	600 000 acre-ft
Median annual flow	2 800 000 acre-ft
Mean annual flow	3 500 000 acre-ft
Upper decile of annual flows	8 700 000 acre-ft
Flow exceeded once in 100 years	18 000 000 acre-ft
Flow exceeded once in 1000 years	32 000 000 acre-ft

DEVELOPMENT OF THE DESIGN CONCEPT

Type of Dam:

During the investigation stage of the project, three main dam types were considered. These were gravity, hollow gravity and rockfill with a clay core. Alternative studies were made and the conclusion reached in 1964 was that a rockfill dam with a sloping clay core and a spillway cut through a low saddle was the most economic and sound engineering solution.

Outlet Works:

The geology and topography of the site dictated that the outlet conduits should be tunnels located in the right abutment. The locations

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Mr Webster is Engineer for Planning Design and Investigation, and Mr Wilkin is Senior Engineer, Major Development Section of the Planning Design and Investigation Branch; both of the Public Works Department, Western Australia.

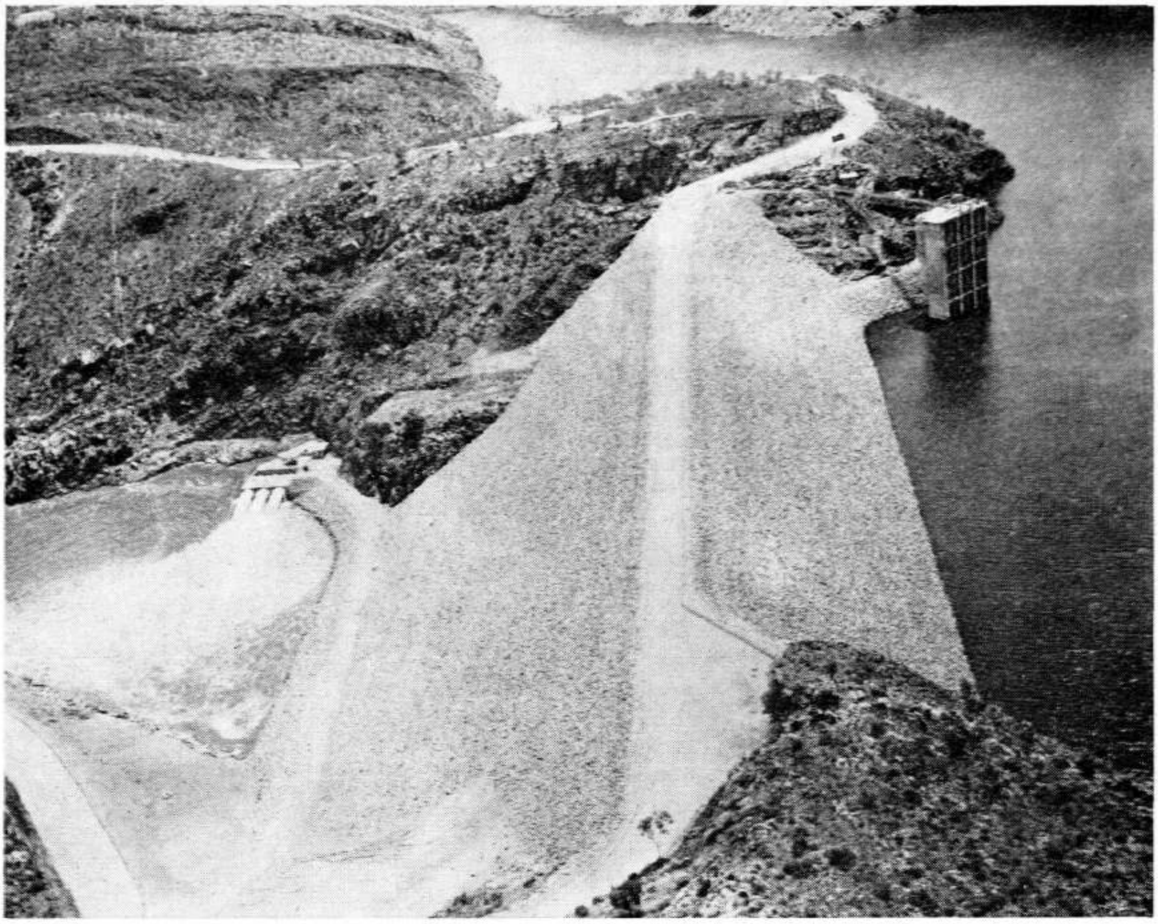

Fig. 1.—View of Completed Dam.

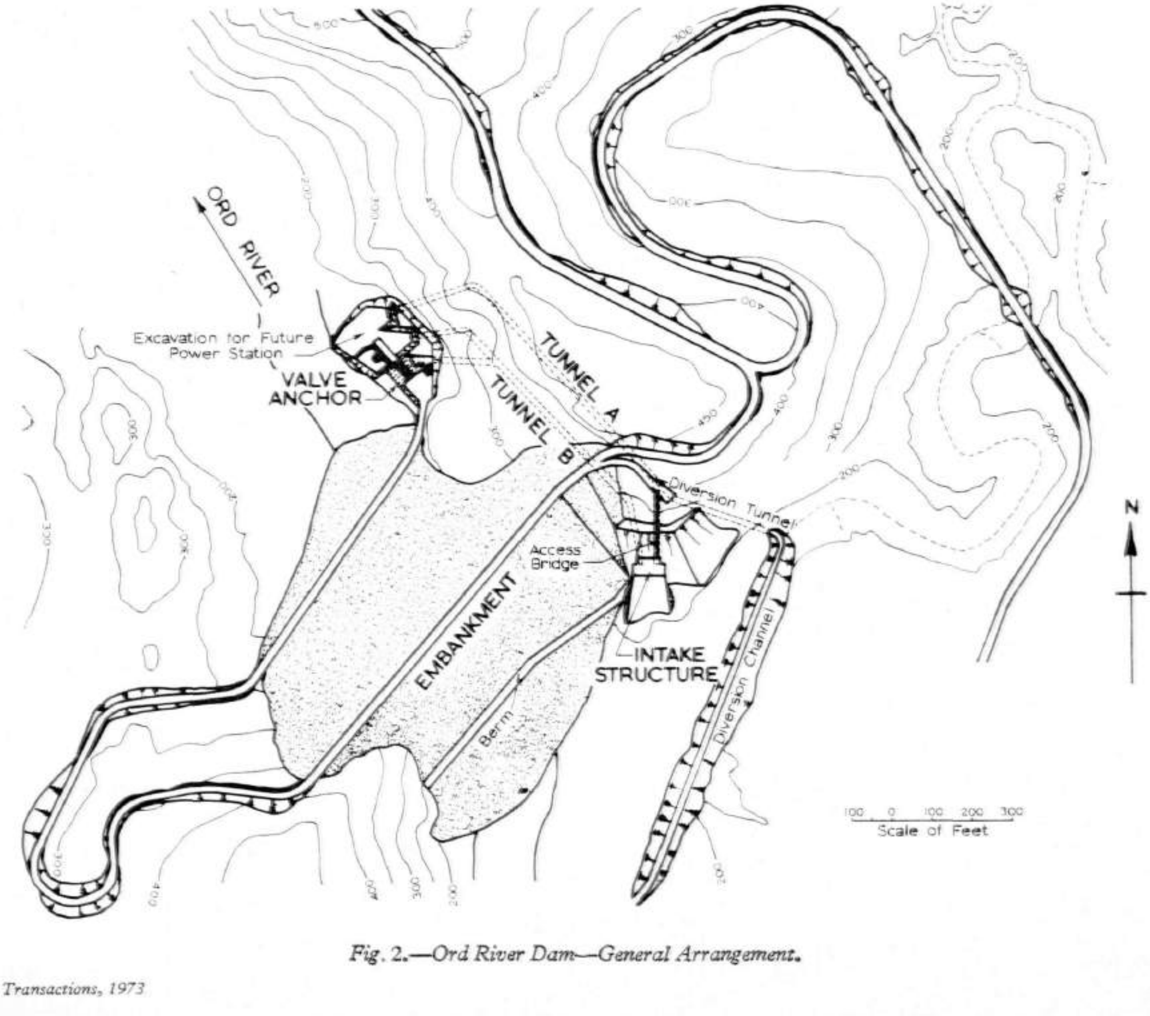

Fig. 2.—Ord River Dam—General Arrangement.

of the intake structure and the outlet valves were also fixed within close limits by the topography.

The system had to provide for irrigation and power requirements and for the diversion of dry season river flows during construction.

Embankment:

Site geology and topography, along with the limitation on the location of the outlet works, were critical in determining the configuration of the embankment. On the right abutment the axis of the embankment was fixed within the limits of these constraints.

On the left abutment the axis of the dam was located so that the core was immediately downstream of the highly jointed quartzite cap and the quartzite-siltstone contact area.

Spillway:

Because of the extremely large flood flows of the Ord River, the spillway played a major part in the investigation and design of the dam.

The concept adopted makes use of the extremely large flood storage volume available. Individual flood events are of reduced importance and the total runoff for a complete season is the governing factor in determining peak spillway discharge. The flood storage will be used to store the runoff from an entire wet season and the flood volume will be discharged at a relatively low rate during that wet season and the subsequent year. With this arrangement, the spillway can be made very small compared with the requirement for the more conventional concept of discharging individual floods. Its cost is thereby very greatly reduced. Against this, the dam embankment has to be constructed somewhat higher in order to provide flood storage, but the spillway costs dominate the economics.

The spillway, which is a narrow channel cut through a saddle in the rim of the reservoir, is some five miles from the dam on a long bay of the reservoir which extends downstream of the dam site. This spillway has a capacity of 125 000 cusecs but in the event of extremely high reservoir levels, three natural saddles will operate as auxiliary spillways, giving a total spillway discharge capacity of approximately 800 000 cusecs at RL 376.

MODEL STUDIES

Embankment:

Placement of the 2 400 000 cu yd of material in the embankment required two dry seasons, necessitating the passage of one wet season's flow over the top of the partially completed dam. Model studies, both two-dimensional and three-dimensional, were carried out to determine stable slopes, type of protection and the height to which the first season's rockfill could be constructed and remain stable when overtopped. These studies showed that the Stage 1 section would be stable if built to RL 160 with 2:1 downstream slope and protected with a 6-ft layer of 3-ft rock.

Spillway:

A model was constructed of the main spillway to test its behaviour. These tests confirmed that the hydraulic calculations were conservative and showed that the assumed normal depth within the channel was not actually achieved as there was accelerating flow throughout the channel length. The studies showed that channel approach banks were required at the entrance to the spillway to improve entry conditions. The shape of these banks and the size of facing rock were determined from the model.

EMBANKMENT MATERIALS

Impervious Core:

Material for the impervious core was a silty-clay of CL classification located approximately a quarter of a mile upstream of the dam on a flat terrace on the right-hand side of the river, as shown in Fig. 4. The properties of this material are: liquid limit 33%, plasticity index 10, optimum moisture content 16.25%. The grading curve for the material is shown in Fig. 5. An area some 3 to 4 miles from the dam site consists of soils derived from the adjacent granite hills, overlying flat ground of weathered in-situ granite. This material, which has a grading as shown in Fig. 5, is a silty-sand of SM classification and was used in the downstream 10 ft of the core zone.

Filter Material:

Filter material was obtained from the river bed deposits upstream from the dam. These sand, gravel and shingle deposits are transported by the river flows and it was realised that the quality and quantity of these deposits could vary considerably from one season to another. It was therefore specified that filter zones should be constructed from material processed so as to fall within the limits shown in Fig. 5.

Rockfill:

Suitable quartzite rock for rockfill was obtained from a quarry located approximately half a mile upstream from the dam on the left side of the river. This quarry site was developed by two coyote blasts, each designed to produce over one million cubic yards of rock. A further source of rockfill was the 600 000 cu yd of granite available from the

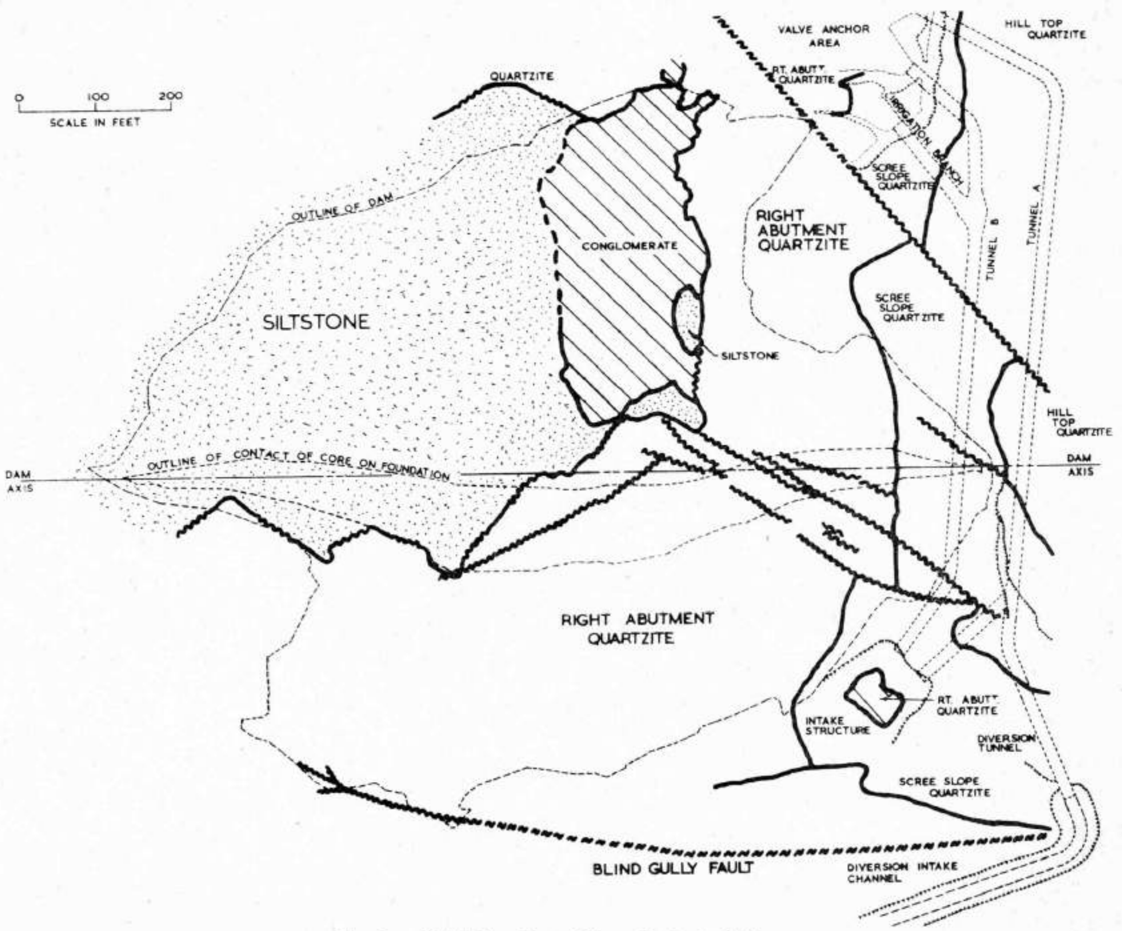

Fig. 3.—Ord River Dam Site—Geological Plan.

excavation for the spillway cut. The use of rockfill from the spillway was left at the option of the contractor, apart from the large-sized rock armouring which was required to be stockpiled as a precaution to ensure that adequate quantities of such material were available for use as protection of the partially completed embankment during overtopping.

EMBANKMENT

Configuration:

Effective use of the available material indicated the adoption of a narrow impervious core. Further, the open-jointed nature of the quartzite on the upstream half of the left abutment restricted considerably the area suitable as a foundation for the core. The minimum thickness of the core was fixed at 0.4 times the maximum head of water, though the infrequency with which higher levels will be reached means that the effective ratio of core width to head will be in excess of 0.5 for nearly all of the time. The 1A Zone of the core is composed of the fine-grained material from the river terrace but on its downstream side there is a 10-ft-wide 1B Zone of the coarser-grained, weathered granite material. The contract provided for two processed filter zones both upstream and downstream of the core. A triangular Zone 3A of rockfill downstream of the core which carries the major thrusts from the water load was required to be placed in 3-ft layers while the remainder of the rockfill, Zone 3B, was placed in 6-ft layers; both zones were compacted by a 10-ton vibrating roller. The whole of the upstream slope and the downstream face below maximum tailwater level have a selected rock rip-rap layer, Zone 3C.

The construction programme necessitated three dry seasons. In the first season the abutments were stripped and the diversion tunnel constructed. In the second season the foundation preparation and the placing of the initial fill were carried out and in the third season, the main body of the embankment was placed.

To protect the initial embankment fill against water flowing over it, the surface between RL 154 and RL 160 was protected by a layer of 3-ft rock. The downstream slope was also protected with a steel grid of 1″-dia reinforcing bars on a 4 ft by 1½ ft spacing anchored into the fill (Fig. 6).

The site conditions resulted in two significant changes being made in the cross-section during construction:

> The inclination of the core was altered by 0.1 to 0.65:1 in an upstream direction in order to avoid the core being draped over a steep cliff face in the river bed which had formed at the quartzite-siltstone contact.
>
> Early testing of the 1B material established that it did not break down during working as allowed for in the filter design. A single downstream filter was therefore substituted between core and rockfill for the original two filters (see Fig. 7).

Embankment Stability:

The stability of the embankment was analysed by the conventional slip circle method, taking account of interslice forces, and modified by the use of tangents at the lower end of the failure surfaces. Check analyses were made at a later stage using a wedge type analysis.

In the complete analyses of failure surfaces carried out using a computer, the factor of safety was defined as the factor by which all the strengths of all the materials in the dam would need to be reduced to bring the dam to the point of failure. This is a more rational definition than the usual one in which it is imagined that all the potential strengths of the materials are developed, an impossible situation.

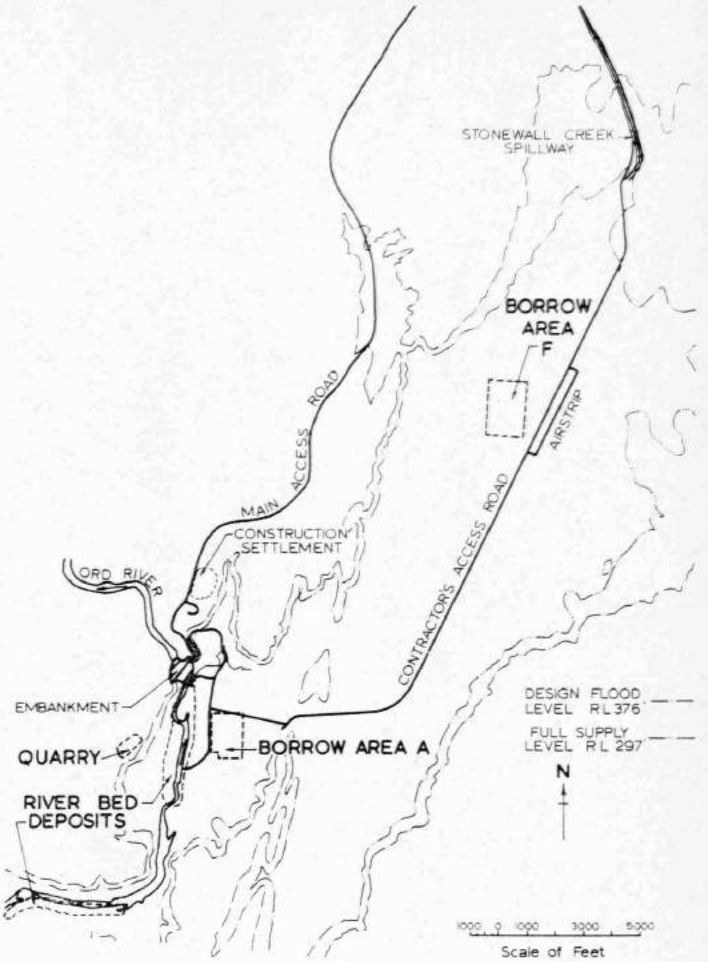

Fig. 4.—Ord Irrigation Project and Ord River Dam—General Plan.

TABLE I

Properties of Embankment Materials Assumed in Design

Property	Zones 1A and 1B	Zones 2A, 2B, 3A, 3B, 3C, 3D
Density	130 lb/cu ft	105 lb/cu ft dry 138 lb/cu ft submerged, total
Friction Angle	11.5°	42°
Cohesion	420 lb/sq ft	0

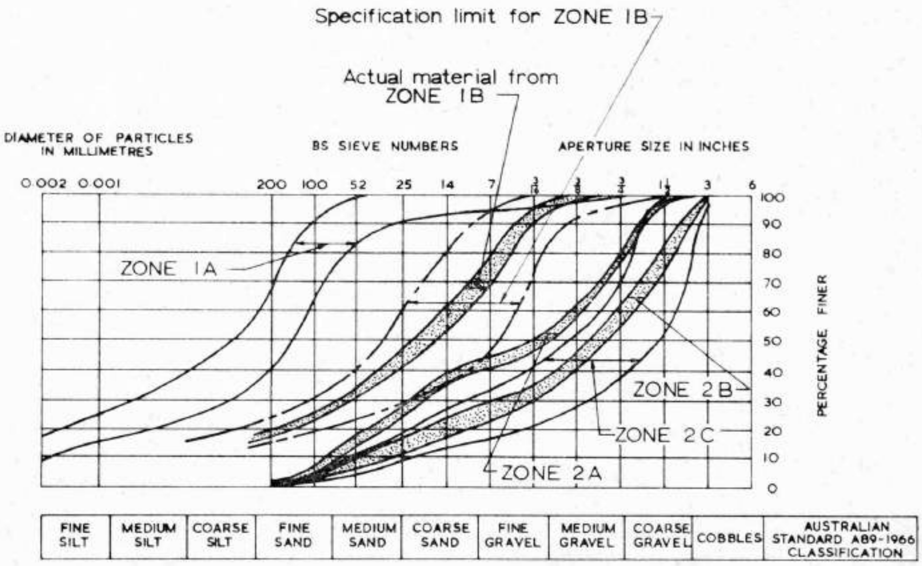

Fig. 5.—Ord River Dam—Properties of Embankment Materials.

Fig. 6.—First Stage of Embankment, showing Protection against Overtopping being placed.

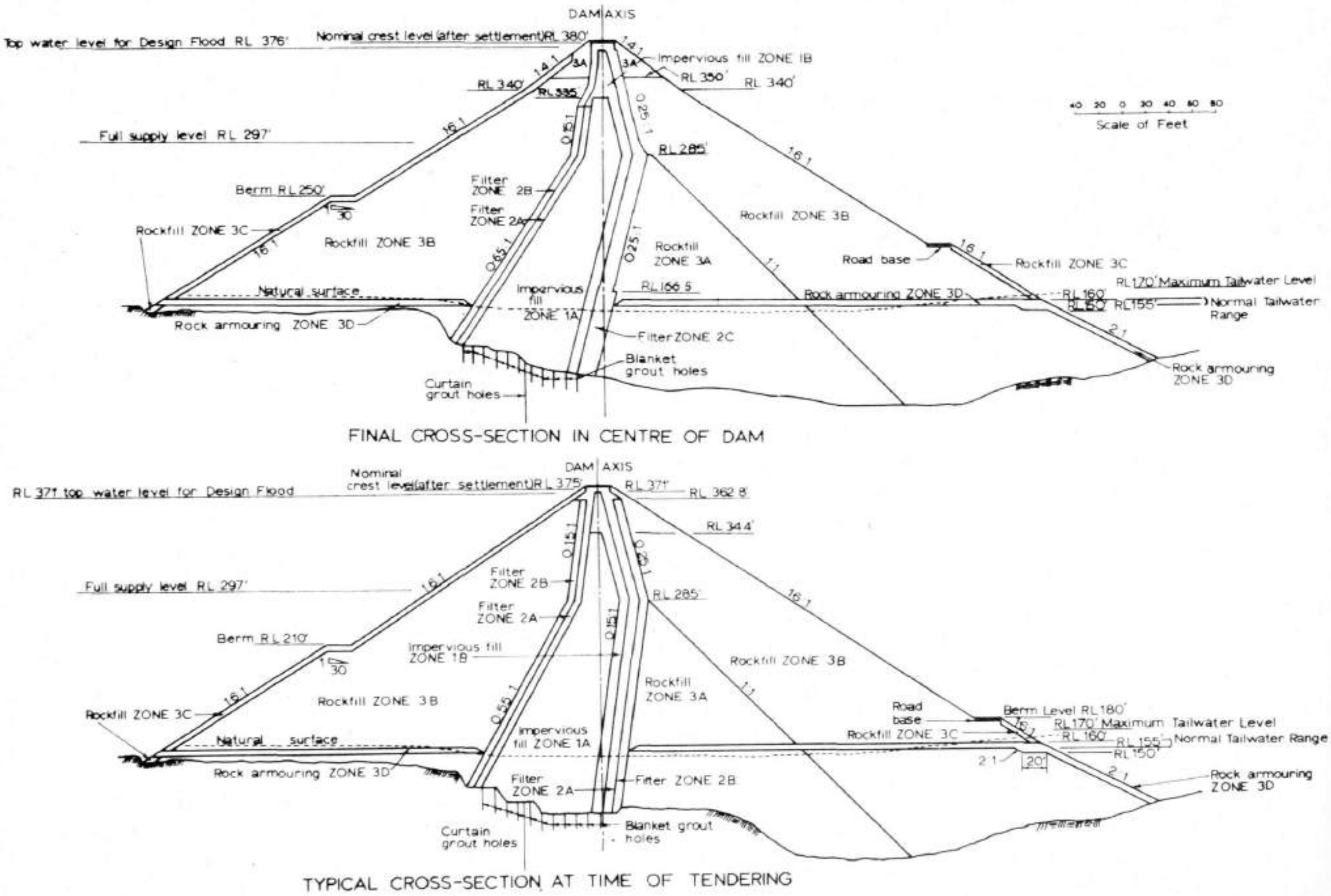

Fig. 7.—Ord River Dam—Embankment Cross-Sections.

The low capacity of the spillway and outlets relative to the very large reservoir capacity makes rapid drawdown an unrealistic design condition. A limited drawdown condition was therefore adopted.

Based on the results of consolidometer tests, design pore pressures, up to 0.55 × height at the deepest part of the critical surface, were used in the analysis, although with the narrow core it was not expected that these pore pressures would be fully developed.

Because of the seismic activity which has been recorded in the Ord area, an earthquake loading, represented by a horizontal acceleration of $0.1g$, was also included in the analysis.

The design factor of safety against design flood level, for failure through the core, was 1.5. The required factor of safety for upstream failure under critical pool or drawdown conditions, was 1.3, and the same value was required for earthquake conditions.

The properties of the materials used in design are shown in Table I.

Foundation Treatment:

In all, 370 000 cu yd of loose material, almost entirely fine sand, were removed from the river bed to obtain a foundation on in-situ rock.

The principal foundation defect was the quartzite-siltstone contact, which crossed under the core at the foot of the left abutment. This zone was excavated, two concrete cut-offs 40 ft apart were constructed to intersect it, and the whole excavation covered with a concrete cap.

As a general protection against seepage through joints close to the surface, which could result in admitting high water pressures to the underside of the core, the core area was treated by blanket grouting, generally with 15-ft-deep holes at 10-ft centres. In a few places, hole spacing was reduced to 5 ft. The actual grout takes were low, with an average take of only 0.18 cu ft per ft of hole. The only area of significantly greater takes was in the weaker quartzite high on the right abutment, where the average take was 0.65 cu ft per ft of hole. A grout curtain, consisting of a single row of holes at 10-ft spacing, up to 100 ft deep with a few holes up to 35 ft deeper, was located approximately under the centre of the core throughout the length of the dam. In twelve places, where grout takes were relatively high, additional holes were drilled which reduce the spacing to 5 ft. This provided a general cut-off against leakage through the foundation. Again, grout takes were low, averaging only 0.20 cu ft per ft of hole.

Instrumentation:

The embankment has been instrumented with a horizontal movement gauge, a vertical settlement gauge, 20 hydrostatic settlement gauges, 36 settlement points and 5 electric piezometers. There are also 3 settlement gauges in the foundation. In the narrow ridge of the right abutment, holes have been drilled for observation of the water table, in case this should affect the stability of the hillside.

To detect possible shocks caused by reservoir filling, an accelerograph is being installed on the top of the embankment and another on the foundation rock. There is a three-motion seismograph at Kununurra.

Silt deposition in the reservoir is to be monitored along 13 contoured strips, by a combination of echo sounding and photogrammetry.

SPILLWAY

The main spillway is cut through a granite saddle and the rock in the centre of the saddle is of good quality, so that no lining is required. Some of the channel at both ends is of less stable material and may suffer some erosion but this will not affect the spillway performance. Control of spillway discharge is determined by the hydraulics of the spillway channel. Rock-faced banks have been provided to improve entry conditions.

Flow from the main and the three auxiliary spillways will be into existing natural creeks which do not have the capacity to discharge spillway flows. It is anticipated that the water will develop these creeks and some substantial movement of material is likely.

Hydrologic Design:

The unusual concept of flood design made it inappropriate to use the conventional " maximum probable flood " method and a probability study of seasonal inflows provided the most satisfactory approach to the design, although some deterministic assessments involving the " maximum probable storm" were also made. For a major structure such as the Ord River Dam, where a failure would be catastrophic, it was considered that the design basis should be for a recurrence interval of not less than 10 000 years plus freeboard.

Using a Monte Carlo process, 50 000 years of monthly rainfalls were generated from 62 years of monthly rainfall data and, using a rainfall/runoff relationship derived from 11 years for which adequate river gauging records were available, monthly runoffs were obtained. The flows of extreme seasons were routed through the reservoir from starting levels determined by first routing the season preceding them in the random sequence. In this way, the recurrence intervals of extreme annual peak water levels were calculated. Because it was found that the distribution of flows within the month had effects on peak levels, random selection from six alternative daily flow patterns were applied to the monthly flows before routing.

The estimated frequencies with which water rises to various levels are shown in Table II.

TABLE II

Frequency of Peak Annual Reservoir Levels

RL	Level exceeded	RL	Level exceeded
297*	9 years in 10	348	1 year in 100
310	1 year in 2	363	1 year in 1 000
326	1 year in 10	372.5	1 year in 10 000 to 15 000

*Spillway Crest.

In most years, the spillway will flow continuously, the median reservoir level at the beginning of the wet season being RL 304 (7 ft above spillway crest). The flood storage at RL 304 is 1 200 000 acre-ft, which is 4% of the total. Following a flood year there will be a greater than median water level when the next wet season arrives. However, it was found that the level to which the reservoir falls by the beginning of the following wet is relatively insensitive to the magnitude of the preceding season's flow. For example, after a 1-in-10-year seasonal flood has occurred, the next season will commence with some 90% of the flood storage still available. This available flood storage reduces to 80% of the total after a 1-in-100-year flood and approximately 72%, or some 20 million acre-ft of the total flood storage, at the start of a season following the 1-in-10 000-year season.

Thus, although the probability analysis took account of this storage " carried over ", it had only a secondary effect on the flood levels at high recurrence intervals. In these circumstances a serious compound effect could only result from the successive occurrence of two seasons so rare that their probability of combined occurrence would be extremely remote and less than once in a million years.

As a further aid to determining the crest level of the embankment, a deterministic assessment was made of the maximum design flood level based on consideration of the maximum probable storm.

The " maximum probable storm " was estimated to have a peak inflow rate of 2 500 000 cusecs and a volume of 15 800 000 acre-ft, i.e. $4\frac{1}{2}$ times the average annual flow or half the volume of the 1-in-10 000-year season discharge. The absolute maximum reservoir level becomes asymptotic at RL 382.5 for a chain of maximum probable floods spaced 15 days apart and for a chain of floods of 80% of the maximum probable, the asymptotic level is RL 380.

At the time of tendering, analysis of all aspects of the spillway hydrology had not been completed, but tendering could not be delayed and the design was issued on the basis of the data then available, the top of the embankment being at RL 375. This was close to the limiting size which could be fitted in the site.

After considering the consequences of dam failure, the possible sources of error in the probability analysis and the small cost of obtaining greater security, a level of RL 380 was adopted for the top of the embankment. This is equivalent to a maximum design level of RL 376 which is estimated to have a recurrence interval of between 30 000 and 50 000 years.

The embankment was modified by steepening the slopes of the upper 35 ft to achieve the extra height. Opportunity was taken to modify the cross-section to simplify placing of the narrow zones near the crest.

Optimisation of Spillway Design:

Two constraints prevented the full optimisation of spillway design, the objective of which was to minimise the total cost of spillway plus embankment. These were the restricted dam site, which limited embankment height, and the desirability of limiting the frequency of operation of the auxiliary spillways.

It was found that the flood storage available was so large that the discharge capacity of the spillway for small and medium surcharges above full supply level had only a minor effect on reservoir operation behaviour and on the maximum flood level reached. Lowest cost was therefore obtained with minimum spillway invert width, and 40 ft was chosen as being a suitable width for the operation of large earthmoving plant. Side slopes of $\frac{1}{2}$:1 were selected for excavation in rock, with a 15-ft-wide berm 50 ft above the invert.

A spillway invert level of RL 297 was adopted so that the auxiliary spillways would not operate more frequently than approximately once in 400 to 600 years, although a higher invert level would have given slightly lower total cost.

OUTLET WORKS

The outlet works were designed to provide for irrigation discharges, for supply to the proposed future hydro-electric station and also to

convey diversion flows during the construction period. The arrangement adopted consisted of two tunnels through the right abutment. One tunnel was located at a low level to provide for diversion of small flows during the dry season and will later be used to supply two 7.5-megawatt sets in the hydro-electric station. The low-level diversion portal was blocked off during the last stages of the construction and permanent flow through this tunnel is by inclined shaft from the intake structure. The second tunnel is also connected to the intake structure by inclined shaft. This tunnel bifurcates at the downstream end to supply both the irrigation outlets and the other two sets of the future hydro-electric power station.

Tunnels:

Each of the tunnels has a finished diameter of 14.5 ft. The criteria used for the design of the tunnels allowed for a partial support of internal pressure by the surrounding rock. The remainder of the support is provided either by reinforcing or by steel lining. The downstream end of the irrigation tunnel and the power station branch of the same tunnel are steel lined for 139 ft and 224 ft respectively. The steel lining, which is fabricated from $\frac{5}{8}$″-plate, commences immediately downstream of the bifurcation, which is constructed in reinforced concrete. The lining is coated with coal tar epoxy applied to a sand-blasted surface.

The upstream portals of the diversion tunnel and both intake tunnels were relocated some 150 ft and 40 ft respectively after the surface scree had been removed from the hillside, to obtain better portalling conditions.

Intake Structure:

A 185-ft-high reinforced concrete intake structure houses two 18 ft by 9 ft fixed-wheel gates to provide for emergency closure and for dewatering the tunnels for inspection and maintenance. The gates are operated by hydraulic hoists connected to the gates by rigid rods. The hoists are located above normal reservoir level, although they will be subject to inundation during extreme floods. Steel trash racks are located upstream of the gates and provision has been made for stop logs. Controls for the gates are located in a machine room at the top of the intake structure and access to the structure is by means of a steel truss bridge from the right abutment to the top deck of the structure. The intake trash racks are protected by zinc metallising and the gates are blast cleaned and coated with a coal tar epoxy.

Outlet Valves:

At the downstream end of the irrigation tunnel, a 14.5-ft-dia steel conduit conveys water to three outlet pipes. These outlets are controlled by Howell-Bunger type regulating valves with discharge hoods to prevent wide cone dispersion, and each regulator is backed up by a butterfly-type guard valve. The guard valves are 96″-dia and the regulating valves 78″-dia. Each outlet has a nominal capacity of 1500 cusecs under minimum head conditions of 77 ft.

Power Supply:

Power supply for gate and valve operation is by separate interchangeable diesel-driven generators housed in buildings adjacent to the intake and valve anchor. The guard valves and fixed-wheel gates can both be closed by gravity without the use of any power.

RIVER DIVERSION

It was impossible to provide full diversion of a river of the magnitude of the Ord in a site as narrow as that in which the Ord River Dam has been constructed. Design of the diversion was based on providing for the exclusion from the dam site of all but extremely infrequent flows during the period from May to October. The capacity of the diversion tunnel was of the order of 3000 cusecs with the upstream and downstream coffer dams at RL 180 and RL 172 respectively.

The final plugging of the diversion tunnel required a rather complex method, made necessary by the diversity of situations which it had to cover.

The design provided for initially dropping a gate at the intake to the diversion tunnel. The rock near the portal is not particularly sound and some leakage into the tunnel even after the gate had closed was to be expected. The tunnel plug is at a distance of 300 ft downstream of the portal, where the rock is much sounder. The closure of the tunnel plug was done in two stages. The bulk of the concrete in the tunnel plug was placed in August, leaving a 4 ft by 4 ft opening through the centre, to allow small flows to pass downstream. Following closure of the portal gate, the final seal was effected by a steel flap gate, which could be closed by remote control, at the upstream end of the opening through the plug. The opening was then plugged with concrete and the joint was pressure grouted.

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Rainfall-Runoff Models Using Digital Computers

BY A. PATTISON, PH.D., M.I.E.AUST. and T. A. MCMAHON, B.E., PH.D., M.I.E.AUST.

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